Literature Review and Study Background

Traffic Load Monitoring and Projections

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1.0 Introduction

This report combines discussions of several issues affecting the performance of this study with summaries of information relating to these issues uncovered in the course of our Task B1 literature review. The report has been divided into three substantive sections plus this introduction. Each of the substantive sections concludes with a list of relevant documents reviewed and incorporates references to these documents (by author and year) in the text of the section.

The first substantive section (Section 2) presents an introduction to several issues relating to the use of traffic data in designing pavements. These relate to the accuracy of the traffic and weight data collected and to the use of this data for estimating current equivalent single axle loads (ESALs) and for forecasting cumulative ESALs. The principal focus of the current study is on the effects of alternative procedures for analyzing traffic and weight data. In order to sharpen that focus, we have been asked to exclude issues of data accuracy from the analyses to be performed in the subsequent tasks of this study. Consistent with this exclusion, we shall also treat ESALs as being defined by the AASHTO equations and tables that relate ESALs to axle weights; our analyses will not address the use of alternative procedures for deriving load equivalency factors from axle weights that are used by some states or that could be developed in the future.

The third section presents a brief overview of other issues relating to pavement design. These generally relate to the equations that are used to design pavements that are expected to last for a specified period of years. Experience has shown that, because of a combination of random and systematic errors in the design process, actual pavement life frequently differs appreciably (in either direction) from its intended life.

Random errors affecting the design process generally relate to difficulties in forecasting cumulative ESALs, difficulties in estimating or measuring any of the other factors (soil characteristics, drainage, climate, etc.) that affect pavement life, and various influences on pavement life that are not fully reflected in the design equations. One type of systematic error stems from systematic errors in estimating the values of any of the variables used by the design equations. Another type (from the standpoint of an individual state or substate region) may result from inaccurate adjustments in these equations for various environmental factors. Other possible sources of systematic error include design equations that have not been adequately modified over the years to reflect the effects of changes in vehicle characteristics or of improved procedures for maintaining pavements (such as improved joint maintenance for rigid jointed pavements).

Sources of systematic errors in the pavement-design process are being identified as part of the Long Term Pavement Performance (LTPP) project and other ongoing research, and modified procedures and equations are being developed to reduce or eliminate these systematic errors. Eventually, the magnitude of these errors should be substantially reduced. For this reason, and also because of the difficulties that would be involved in analyzing the effects of these errors, we plan to exclude from our analyses all systematic errors except for those affecting the estimation and forecasting of cumulative ESALs; also, as indicated above, we plan to exclude the systematic effects of the use of alternative equivalency factor equations or tables. Thus, in this study, we will limit our analyses of the effects on life-cycle pavement costs to effects that relate to the procedures used for developing forecasts of cumulative ESALs (as defined by AASHTO) and to non-traffic-related random influences on pavement design and pavement life.

The final section of this report discusses general procedures for analyzing the life-cycle costs of pavements as well as specific issues relating to the use of these procedures.

2.0 Forecasting Cumulative ESALs

For existing roads, the development of forecasts of cumulative ESALs over the design life of a pavement usually is divided into three major steps:

1. Estimation of current annual average daily traffic (AADT) by vehicle class (VC);

2. Conversion of these estimates to estimate of current (annual or average daily) ESALs in the design lane; and

3. Forecasting changes (growth) in the rate of ESALs application over the design life of the pavement.

For new roads, current traffic data does not exist. Accordingly some form of special study, possibly using a travel-demand forecasting model, is required to estimate use of the new road (FHWA, 1988). Such studies may also be performed when major capacity increases are expected to have a significant effect on use of existing roads. Forecasts of the use of new roads and of roads undergoing major capacity increases are inherently less accurate than forecasts of the use of existing roads that are not undergoing major capacity increases.

In the current study, we will analyze only the life-cycle cost (LCC) effects of alternative procedures for forecasting cumulative ESALs for existing roads that are not undergoing major capacity increases. The three major steps in the development of cumulative ESALs forecasts are discussed in the three subsections below. The third of these subsections also includes some brief comments on the use of travel-demand forecasting models for forecasting truck traffic.

Estimating AADT by Vehicle Class

There are several different procedures that are or can be used for estimating AADT by VC for a section of road. These include:

1. *Use of "Typical" VC Distributions*. A procedure that minimizes data requirements involves estimating AADT (usually by applying seasonal and day-of-week (DOW) factors to a short-duration count of traffic volume), and then distributing this estimate across vehicle classes using classification data obtained from one or more sites that are believed to carry traffic with VC distributions that are similar to traffic on the segment in question.

2. *Unadjusted Short-Duration Classification Counts.* One alternative to the above procedure is to obtain a set of short-duration classification counts on the segment in question and to use these counts without any adjustment for seasonal and DOW traffic volumes. Most frequently these counts are obtained for a 48-hour weekday period, in which case the estimates of AADT by VC are obtained by dividing the counts by two.

3. *Distribution of AADT Across VCs.* Another alternative is to estimate total AADT by applying seasonal and DOW factors to a count of total traffic on the segment in question and then to use a set of short-duration classification counts (usually obtained at the same time as the volume count) to distribute the estimate of total AADT across VCs. The principal problem with this procedure is that, on most segments, truck traffic drops appreciably on weekends and automobile traffic does not. Hence, the use of weekday classification counts in this procedure produces a significant upward bias in the resulting estimates of truck AADT (Weinblatt, 1996). For many segments on which total traffic rises on weekends, the upward bias is actually greater than it is for Procedure 2.

4. *Volume Factors.* A variant of Procedure 3 is to apply the seasonal and DOW factors (obtained from volume data) directly to the short-duration classification counts. In this procedure, the *same* factor is applied to each of these counts. Since the same factor is applied to each count, for any pair of classes, the ratio between their estimated AADTs is the same as the ratio between their original counts. Similarly, the ratio of truck AADT to total AADT is the same as the ratio of the original count of trucks to the original count of total vehicles. This procedure produces the same estimates of AADT by VC as does Procedure 3 and has the same disadvantages as that procedure.

5. *Factoring by Class.* A common variant of the above procedure that produces appreciably better results uses separate sets of factors for each of several sets of vehicle classes. This procedure factors truck counts with seasonal and day-of-week factors that are obtained entirely from data for trucks. Hence, the factoring adjusts the raw truck counts for the decline in traffic volume that generally occurs on weekends, avoiding the upward bias produced by the preceding procedure (Weinblatt, 1996).

6. *Distributions from Nearby Continuous Classification Sites.* A variant of Procedure 1 uses estimates of AADT by VC obtained at one or more continuous classification sites (CCSs) to obtain distributions of AADT across VCs, and applies these distributions to estimates of total AADT obtained at nearby sites on the same road. Restricting this procedure to the use of classification data obtained only at reasonably nearby sites on the same road results in relatively high quality estimates of AADT by VC. This procedure is used on the Interstate System (IS) by the Virginia Department of Transportation (DOT) (Cambridge Systematics, 1995).

7. *Using Factors by Class from Nearby CCSs.* In the case of sites that are reasonably close to a CCS on the same road, another alternative is to use seasonal and day-of-week factors for trucks from the CCS to adjust short-duration classification

counts obtained at these sites. (This procedure differs from Procedure 5 in that the factors are obtained from a single nearby CCS on the same road, and it differs from Procedure 6 in that the short-duration counts are classification counts rather than volume counts). This procedure generally will produce somewhat better estimates than Procedure 6.

8. *Seven-Day Classification Counts.* The best way of minimizing errors introduced by day-of-week variations is to perform all classification counting for seven-day periods (Hallenbeck, 1996). Seven-day classification counts may be used in unfactored form, or they may be adjusted using seasonal factors obtained for sets of vehicle classes that distinguish (at least) two-axle vehicles from trucks with three or more axles.[1] A more costly option, recommended by Hallenbeck, et. al., (1997a), is to obtain seven-day classification counts at three or four-month intervals and to average the results without factoring.

A recent survey (Stamatiadis and Allen, 1997) indicates that about two-thirds of the states do not apply seasonal adjustments to vehicle classification data and that, of those that do, only one state specifically indicated that truck counts are adjusted using factors derived from truck data (Procedure 5) rather than from data for total traffic volume (Procedure 4). Thus, it appears that most states currently use one of the first five of the above procedures, with Procedures 2 and 4 likely being the most common.

Of the first five procedures, the fifth is clearly the best. However, even for this procedure, the quality of the estimates of AADT by VC is dependent on the similarity between the DOW and seasonal patterns of truck traffic at the site in question and the corresponding patterns at the CCSs in the factor group(s) to which the site is assigned. Hallenbeck, et. al (1997c) observe that the factor groups used for factoring truck traffic generally should not be the same as those used for total volume. They also observe that the degree of time-of-day variation in truck traffic can be used as an indicator of the percentage of nonlocal truck traffic in the traffic stream, a significant influence on DOW patterns. A moderate degree of error in the estimates produced by this procedure is unavoidable: a review of several studies of the effectiveness of factoring procedures for estimating *total* AADT (Davis, 1997) indicates that root-mean-square percentage errors in these estimates are unlikely to be less than seven percent; and the smaller volumes observed when counting traffic for individual VCs inevitably results in higher percentage errors (Wright, et. al. 1997).

Procedures 7 and 8 appear to be better procedures for pavement-design purposes than any of the others, and it is expected that the next edition of FHWA's *Traffic Monitoring Guide* (TMG) will encourage the use of some form of these procedures for this purpose. Procedure 5 is likely to be recommended as a basic procedure

[1] One option is to use the day-of-week patterns from the seven-day count as an aid in assigning the count site to a factor group, as is done in Zurich (Jarema, et. al., 1997). However, it is unclear how useful this step is for seasonal factoring, since seasonal and day-of-week usage patterns are not necessarily correlated.

for estimating overall truck VMT, and Procedure 4 is expected to be rejected because of the biased estimates that it produces.

The Work Plan for our current project calls for an analysis of the LCC benefits of using Procedure 5 for pavement-design purposes instead of Procedures 1, 2 or 4. This analysis will build upon a substantial amount of information that is already known about the performance of these procedures. **More interesting results could be obtained if the focus were shifted to a comparison of Procedures 5 and 8;** however, any analysis of the effectiveness of Procedure 8 would require a substantial reallocation of study resources because much less is known about its performance than about that of Procedures 1 - 5.

Our analyses will assume that all classification counts are collected in both directions, thus avoiding issues relating to traffic volumes that are not balanced on individual days of the week (Wright, et. al., 1997, Table 3) and relating to the use of lift axles and logging-truck dollies that result in vehicles that operate with more axles in the loaded direction than in the empty direction. Our analyses also will not directly address the effects of bad classification counts, a particular problem for Class 5 (six tire) trucks which are frequently undercounted by 50 percent or more (Harvey, et. al., Tables 4, 7, 13, 14, 17, 21, 24, 27, 30, 32, 35 and 38).

Estimating ESALs

Estimates of daily average ESALs on a road segment are developed by obtaining estimates of AADT by VC and multiplying by corresponding estimates of average ESALs per vehicle for each VC. In addition to varying by VC, average ESALs per vehicle may vary by DOW, season, road or road segment, direction, and lane. As stated in Section 1, for the purpose of this study, we shall treat ESALs as being defined by the AASHTO equations and tables that relate ESALs to axle weight; we will not address the use of alternative procedures for deriving load equivalency factors from axle weights that are used by some states or that could be developed in the future.

The TMG (FHWA, 1995) requires each state to collect WIM data from at least 90 sites to derive estimates of average ESALs per vehicle by VC for IS and non-IS roads. Most states also use this data (or just the data collected from continuously operated WIM sites) to develop estimates of average ESALs per vehicle by VC (possibly by class of road) for use in estimating total ESALs at sites of interest. However, use of these average values ignores the very significant differences that can exist at different sites. Average ESALs per Class 9 truck have been found to differ by as much as a factor of eight at two LTPP sites in Washington State, leading LTPP to issue a TechBrief emphasizing the importance of using site-specific data (FHWA, 1998b).

It has been suggested (Hallenbeck, et. al., 1997a) that estimates of ESALs per vehicle be derived from WIM data collected on segments of interest during a seven-day period. The use of a seven-day period provides for automatic incorporation of DOW variation in ESALs per vehicle – Hallenbeck and Cornell-Martinez (1998) estimate that these values go up on weekends at about 18 percent of all sites and go down on weekends at the same percentage of sites. However, it does not incorporate seasonal variation, which was found to exist but to be less significant than DOW variation; if time and resources permit, Hallenbeck, et. al. (1997a), suggest collecting additional ESALs data for seven-day periods at other times of the year. Even with these procedures for using site-specific data, great accuracy cannot be expected in the resulting estimates of total ESALs; Hallenbeck, et. al. (1997b), have estimated that the use of one week of classification counts and one week of WIM data (without seasonal adjustments) produces ESALs estimates with an expected mean error of ±30 percent and with 95 percent confidence that the error is less than ±80 percent.

The American Association of Highway and Transportation Officials (AASHTO) Guide (1993, Appendix D) shows samples of ESALs estimation using 14 distinct vehicle classes which are also grouped into five broader classes, and FHWA's TMG requires ESALs estimates for 10 truck classes. However, the use of a large number of classes requires the estimation of AADT and ESALs per vehicle for some relatively uncommon classes for which neither average volumes nor average ESALs can be estimated accurately. Accordingly, for the purpose of estimating total ESALs, vehicle classes are frequently aggregated. One common aggregation is: single-unit trucks (FHWA Classes 5 - 7), single trailer trucks, and multi-trailer trucks.

A related issue is the treatment of two-axle (four and six-tire) trucks since (as discussed in the preceding section) counts of these vehicles made with automatic vehicle classifiers can be quite inaccurate. Other issues affecting the estimation of ESALs include maintaining the calibration of permanent WIM scales (FHWA, 1998a), and the upward bias in ESALs values produced when dynamic weight data is used from scales that are calibrated to static weights on the basis of weight rather than on the basis of ESALs. (This bias occurs because the mean of the fourth power of a set of scale readings is higher than the fourth power of the mean.)

Site-specific data obtained by direction provides for appropriate reflection of directional differences in ESALs per vehicle; these differences can be appreciable on road segments where significant numbers of trucks operate loaded in a particular direction and unloaded in the opposite direction.

For multi-lane roads, site-specific data also can provide information on lane usage (reflecting variations that occur in vehicles by class and in ESALs per vehicle) *at the site*; however, since lane usage varies over the course of any road segment (as a result of changes in grade and other factors), this information may not be representative for an entire road segment. Instead, lane distribution factors (representing estimates of the percentage of total ESALs in the design lane) are usually used. The factors suggested by AASHTO (1993) are shown in Table 2.1

Table 2.1 Suggested Lane Distribution Factors for ESALs

Number of Lanes	Percent of ESALs

in One Direction	in Design Lane
1	100
2	80 - 100
3	60 - 80
4	50 - 75

Source: AASHTO, 1993, p. II-9.

The Work Plan for our current project calls for analyses of the LCC benefits of using site-specific estimates of ESALs per vehicle instead of estimates from other sites. The Work Plan for Task C also calls for analyses of the effects on LCC costs of typical seasonal variations in ESALs per vehicle when these variations are not incorporated into the estimates that are used, and also the effects of differences between actual and assumed lane-load distribution factors. Our analyses will not address the effects of other issues affecting the accuracy of estimates of ESALs per vehicle.

Forecasting ESALs

Changes (usually increases) in annual ESALs on any road segment may result from changes in:

1. Total traffic on the segment;

2. The percentages of vehicles in each of the vehicle classes distinguished in the ESALs estimation process; and

3. Average ESALs per vehicle for each of these classes.

The distinction between Numbers 1 and 2 usually is necessary when travel-demand forecasting models are used, but it is less helpful otherwise. The influences on truck traffic are somewhat different than those on automobile traffic, and there is limited need to address the latter influences when one is principally concerned with changes in truck traffic. On most roads, truck traffic has grown faster than automobile traffic and combination trucks have grown faster than single unit trucks,[2] and these differences in growth rates are likely to continue.

In general, forecasts of truck volume are developed by observing past trends and identifying any reasons for expecting major changes in these trends. On roads that carry truck traffic with origins or destinations that are relatively nearby, truck

[2] Nationally, vehicle-miles traveled by combination trucks grew between 1980 and 1996 at a compound annual rate of 3.7 percent, while the corresponding rates for single-unit trucks and for other vehicles were 3.25 and 3.1 percent, respectively (FHWA, 1997a and 1997b).

volumes may be influenced significantly by a few major operators of truck traffic; for these roads, the trends in truck traffic should be adjusted to reflect likely changes in the rates of expansion or contraction of activity at these traffic generators as well as possible future truck traffic produced by new traffic generators. On roads carrying large numbers of trucks with more distant origins and destinations, traffic trends should be adjusted for the effects of the planned development of alternate routes.

The literature generally recommends that separate growth rates be developed for two or three groups of truck classes, but there is no consensus as to what these groups should be. AASHTO (1993, Appendix D) presents two examples in which three separate growth rates are used: for single-unit trucks; for single-trailer combination trucks; and for multi-trailer combinations. Using separate growth rates for these three groups of VCs, or at least for single-unit trucks and combination trucks, appears to be appropriate; however, AASHTO does not provide an explicit recommendation to this effect. Any set of VC groupings that are used for this purpose either should be the same as those used in the analyses of AADT by vehicle class and average ESALs per vehicle, or they should represent aggregations of the vehicle classes used in these analyses.

Travel-demand forecasting models are necessary for forecasting usage of a new facility for which historical data are not available, and many Metropolitan Planning Organizations (MPOs) use these models routinely for generating traffic forecasts. However, for the purpose of forecasting truck traffic, these models have limitations. Most are designed only to analyze personal travel, so some further analysis is required to derive forecasts of changes in truck volumes. At a minimum, the forecasts must be adjusted to reflect the tendency for truck traffic to grow faster (or to decline more slowly) than total traffic. A further consideration is that truck volumes usually are low during peak periods for passenger travel, so daily truck traffic is likely to be much less affected by peak-hour congestion than is passenger-vehicle traffic; accordingly, the forecasts produced by these models of the adaptation of urban traffic to increasing congestion are unlikely to hold for trucks.

As observed at the beginning of this subsection, in addition to forecasting changes in AADT by VC for each of the aggregate vehicle classes distinguished in the ESALs estimation process, it is also necessary to forecast changes in average ESALs per vehicle for each of these classes. These changes may result from:

- A shift from one narrowly defined class to another in the same aggregate vehicle class (e.g., a shift from the use of FHWA Class 8 vehicles to Class 9 will result in increasing average ESALs per vehicle for the aggregate class of combinations);

- Changes in the loading capacity of vehicles in a given FHWA VC; and

- Changes in the average density of commodities carried by vehicles in a given FHWA VC or in the percentage of vehicles that are empty.

The first two types of change occur primarily because of changes in truck size and weight limits. Although changes in these limits are difficult to forecast, it is likely that such changes will continue to occur and will cause modest increases in average ESALs per vehicle (Battelle Team, 1995). One paper (Backlund and Gruver, 1990) suggests forecasting such increases on the basis of past trends if trend data are available, and otherwise assuming a one-percent average annual increase for combinations with five or more axles and no increase for smaller vehicles.

The third type of change occurs primarily because of changes in the mix of commodities carried. If there is reason to believe that transport of major weight-limited bulk commodities (grain, minerals, logs, construction materials, etc.) will grow at a significantly different rate than transport of other commodities, it may be worthwhile to develop separate forecasts for the two types of commodities and to adjust the forecasts of average ESALs per vehicle accordingly.

The Work Plan for our current project calls for analyses of the LCC effects of:

- Using forecasts of growth in truck volumes rather than growth in total traffic; and

- Using site-specific forecasts instead of statewide forecasts which may over or under-estimate growth on the road in question.

The Work Plan also calls for analyses of the LCC effects of assuming one type of growth trajectory for a road on which growth actually follows a different trajectory (e.g., exponential growth instead of linear growth).

Our analyses will assume that future growth in traffic will conform to past trends and that, as Backlund and Gruver suggest, there will be an additional one percent annual increase in average ESALs per vehicle for combinations (but no increase for single-unit trucks). We do not plan to analyze the LCC effects of future growth rates that differ from past growth rates.

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3.0 Other Factors Affecting Pavement Design

This section presents some brief background material relating to nontraffic factors affecting the design and performance of pavements, with a primary focus on factors affecting the analyses to be performed in subsequent tasks of this study. Since our analyses will be limited to pavements designed using the 1986 and 1993 AASHTO procedures and, at least initially, to rigid pavements and to flexible overlays of rigid pavements, the material presented in this section relates primarily to the AASHTO procedure for designing rigid pavement.

The first subsection below presents a brief summary of the non-traffic factors affecting pavement deterioration and the related variables used by the AASHTO procedure for designing rigid pavements. The second subsection discusses the use of a "reliability design factor" in AASHTO's procedures in order to increase expected pavement life and reduce the probability of premature failure. The final subsection contains a very brief discussion of a complex subject – limitations of the AASHTO procedures, tendencies of these procedures to produce consistent overdesigns or underdesigns of pavement in some parts of the country, and current research into potential modifications that will reduce or eliminate these tendencies. As stated in Section 1, we plan to exclude from our analyses the effects of systematic errors produced by the use of the AASHTO equations but not the effects of random errors.

Nontraffic Factors Affecting the Design and Performance of Pavements

The goal of pavement design is to design pavement which, given the stresses to which it is subjected, is expected to remain in serviceable condition for some specified minimum period, or service life. During this period, the pavement will deteriorate at a rate that varies with condition and is also influenced by a variety of other factors. These include:

- The pavement design;
- Stresses from traffic, generally measured as cumulative ESALs;
- Subgrade support;
- Subsurface moisture;
- Temperature fluctuations;

- Aging of pavement materials; and
- Solar exposure.

The procedures most commonly used in the United States for designing pavements are those originally presented in the 1986 AASHTO Design Guide and revised in the 1993 AASHTO Design Guide (AASHTO, 1986 and 1993). However, as of 1995, only 35 states were using either the 1986 or 1993 AASHTO procedures for designing rigid pavements and only 25 were using the procedures for designing flexible pavements (Von Quintus and Killingsworth, 1998). Earlier AASHTO procedures, from 1972, were being used for designing rigid pavements by seven states and for designing flexible pavements by 14 states; while a variety of other procedures were being used by the remaining states. The procedures used in Canada differ from the AASHTO procedures in that, among other things, load equivalency factors increase with the square or cube of axle load instead of with roughly the fourth power of axle load as in the case of the AASHTO ESALs equation (Haas and Kazmierowski, 1997).

In this study, we will focus almost exclusively on the 1993 AASHTO procedures (which we shall refer to simply as "the AASHTO procedures"). Variables used by the AASHTO procedures for designing and forecasting the performance of rigid pavements include:

- Initial serviceability index (p_1);
- Terminal serviceability index (p_t);
- Surface slab thickness (D);
- Surface rupture modulus (S'_c);
- Surface load transfer factor (J);
- Drainage factor (C_d);
- Slab elastic modulus (E_c);
- Effective subgrade reaction modulus (k); and
- Predicted ESALs over the design period (w_{18}).

The terminal serviceability index is set by the highway agency, usually to in the 2.0 to 3.0 range, frequently varying by functional system. All the other variables in this list are measured or estimated, and there is some uncertainty in the values selected for each variable. Partly for this reason, pavement condition generally deteriorates at a rate that differs from that predicted by the AASHTO equations, and the time for pavements to reach terminal serviceability generally differs from the design life of the pavement.

Hughes (1996, Tables 25 and 26) reproduces 14 summaries of data from four states relating the design thickness (D) of Portland cement concrete (PCC) pavement to actual thickness measured from core samples and showing the standard deviations and coefficients of variation of the measured thickness. The data indicate that measured thickness was 0.8 to 5.5 percent greater than design thickness, and the coefficients of variation (the standard deviations divided by the corresponding means) were between 1.9 and 5.4 percent. Overall, measured thickness was, on average, 3.2 percent larger than design thickness, and the average coefficient of variation was 3.4 percent.

A recent survey (Jiang, Darter and Owusu-Antwi, 1996) identified values commonly used by the states for several of the above variables. Table 3.1 summarizes the means of the values currently used.

Table 3.1 Mean Values Used for Variables Required for Designing Rigid Pavements

Design Variable			**Mean Value**
$p_1 - p_t$	Change in serviceability		1.9
J	Surface load transfer factor	Doweled JPCP	2.8
		Doweled JRCP	2.9
		CRCP	2.6
		Nondoweled JPCP	3.8
C_d	Drainage factor	Wet	1.11
		Dry	1.22
k	Effective subgrade reaction modulus	Soft soil	75
		Medium soil	150
		Stiff soil	300

Source: Jiang, Darter and Owusu-Antwi, 1996.

The Reliability of Pavement Designs[3]

AASHTO introduces the concept of the *reliability level,* R, of a pavement design, defining this quantity to represent the probability that pavement lasts at least as long as its design life. Recommended reliability levels vary with functional system. The levels recommended by AASHTO are shown in Table 3.2.

Table 3.2 Reliability Levels Recommended by AASHTO

[3] Most of this section is a highly simplified summary of material from Part I, Chapter 4 of the AASHTO Guide (1993).

Functional Class	Urban	Rural
Interstate and Other Freeway	85-99.9%	80-99.9%
Principal Arterial	80-99	75-95
Collector	80-95	75-95
Local	50-80	50-80

Source: AASHTO, 1993, Table 2.2, page II-9.

In order to achieve any given reliability level greater than 50 percent, the AASHTO procedure uses a modified equation for forecasting pavement deterioration that incorporates an extra term allowing for some increased deterioration above that which is expected. One form of this extra term is $-\log_{10} F_R$, where F_R is the *reliability design factor.*

In order to obtain an operational definition of F_R (or log F_R), AASHTO observes that it is reasonable to approximate the distribution of the logarithm of the life of a pavement with a normal distribution. The mean of this distribution is the logarithm of the expected value of the pavement life, and the variance is identified as S_o^2. AASHTO then shows that, with this approximation,

$$\log_{10} F_R = Z_R \times S_o \tag{3.1}$$

$$(\text{or } F_R = 10^{-Z_R x S_o})$$

where Z_R is the standard normal deviate. Selected values for Z_R are shown in Table 3.3.

Table 3.3 Z_R Values Corresponding to Selected Levels of Reliability

Reliability (R) (percent)	Z_R
50	-0.000
75	-0.674
80	-0.841
85	-1.037
90	-1.282
95	-1.645
98	-2.054
99	-2.327

Source: AASHTO, 1993, Table 4.1, page I-62.

Using Equation 3.1, the extra term in the AASHTO pavement deterioration equation becomes $Z_R \times S_o$. The size of this term thus varies with both the level of reliability required (dropping to zero at 50 percent reliability) and the square root of the variance, S_o^2, of the distribution of the logarithm of pavement life. The latter variance, in turn, can be treated as the sum of two separate variances:

S_w^2 = the variance of the distribution of the logarithm of predicted design period ESALs; and

S_N^2 = the variance of the distribution of the logarithm of "pavement performance" (i.e., pavement changes resulting from a given set of ESAL applications).

AASHTO estimates[4] that the latter source of variance is responsible for 73 percent of total variance for rigid pavements and for 82 percent of total variance for flexible pavements, and that significant shares of this variance are due to a lack of fit in the pavement performance equation and to unidentified variables. In the case of rigid pavements, the variables with the largest identified contributions to S_N^2 are said to be variances in the surface rupture modulus (S_c), the drainage factor (C_2), and slab thickness (D). However, the interpretation of this information is difficult because AASHTO is unclear about the set of observations used in estimating these variances. For example, it is not clear whether the variance for the drainage factor reflects a national variance or a variance for a particular climate zone. (The variance for a particular climate zone would be smaller than a national variance, and the variance for a dry zone probably would be smaller than that for a wet zone.)

Effectiveness of the AASHTO Procedures

In this subsection we present a very brief review of some information about the quality of the pavement designs produced by the AASHTO procedures and of some of the current work being done to improve the procedure. This review is in no way a comprehensive review of these issues, which would be beyond the scope of our present effort. However, the review should be useful in providing some understanding of limitations of the current procedures. For reasons discussed in Section 1, we do not plan to analyze the systematic effects on life-cycle pavement costs of the limitations discussed in this section.

The AASHTO procedures were originally developed using the results of the AASHO Road Test conducted in Ottawa, IL, in 1958-1960. The procedures are subject to several limitations, such as (Chen, Bendaña and McAuliffe, 1995, pp. 49-50):

[4] AASHTO, 1986, Appendix EE, Tables EE.4 and EE.5.

- Since the Road Test lasted only two years, damage due to aging is not reflected in the procedures.

- The Road Test results are completely valid only within the experimental range of the test (maximum cumulative ESALs of about 10 million, maximum thickness of flexible pavement of only six inches, and only one soil type).

- An oversimplifed linear relationship between structural number, thickness, and structural coefficient of each component layer is used.

- Present Serviceability Index (PSI) is a unidimensional index of functional performance which emphasizes roughness rather than structural distresses such as cracking or rutting.

Additional limitations of the AASHTO procedure for designing rigid pavements as well as several suggested improvements to the procedure are discussed by Darter, Owusu-Antwi and Ahmad (1996).

Several studies have been conducted in order to produce modifications to the AASHTO procedures that allow them to be used outside their original experimental range, particularly in areas with different soil and climate conditions. However, despite these efforts, some states have found limitations in the ability of the AASHTO equations to estimate pavement performance in their states.

One recent study (Mukhtar and Abdulshafi, 1996) conducted in Ohio found that flexible pavements deteriorated appreciably more slowly with ESALs than would be predicted by the AASHTO equation. For rigid pavements, this study found no significant differences between actual deterioration and that predicted by the AASHTO equation in the first few years of pavement life; however, the limited age of the rigid pavements evaluated prevented any clear conclusion from the review of data for rigid pavements.

Appreciably different results were obtained in a recent study conducted by New York State (Chen, Bendaña and McAuliffe, 1995, and Chen, et. al., 1996). In the case of flexible pavements, significant differences in the pavement designs produced by the AASHTO and New York State procedures were found only on roads with high traffic volumes; and, for these roads, it appears that reducing pavement thicknesses somewhat, as suggested by the AASHTO procedure, would not reduce pavement lives excessively. (The New York State pavements tended to fail because of aging and environmental factors, rather than because of structural deterioration due to traffic.)

On the other hand, for rigid pavements, the New York State study found that the AASHTO equation substantially underpredicted pavement life. This result was provisionally attributed to improvements in joint maintenance that have been developed since the original AASHO road tests were conducted. To reflect the effects of improved joint maintenance, the New York State researchers have developed and adopted a modified version of the AASHTO equation.

Additional efforts to evaluate and improve upon the AASHTO equations continue as part of the Long Term Pavement Project (LTPP). One major effort developed improved procedures for obtaining values for several of the parameters required by the AASHTO procedures, examined the applicability of the drainage coefficients and relative damage factors used by these procedures, and considered the effects on flexible pavements of seasonal variations in material properties (Von Quintus and Killingsworth, 1998). The results of this study include better procedures for estimating parameters (published separately in three FHWA Design Pamphlets). However, the study also found that some aspects of the AASHTO procedures, such as the drainage coefficients could not be substantiated. It appears that further research is needed to determine whether additional modifications to the AASHTO procedures are required.

As stated in Section 1, for the purpose of our current study, we intend to ignore the effects of systematic errors produced by the current AASHTO procedure. One reason for this decision is that the procedure is evolving and the quality of the pavement designs it produces can be expected to improve as a result of ongoing LTPP research. Another is that any state that observes systematic errors in the results produced by the procedure for the entire state or for any substate region can modify the procedure to eliminate the systematic errors. (Such modifications generally will introduce an additional source of random errors, but, on average, these will be appreciably smaller than the systematic errors.)

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4.0 Life-Cycle Costs

Pavement-life cycle costs (LCCs) consist of all net costs resulting from the provision of a pavement during its life. These include costs to the highway agency, costs and benefits to the highway user, and external costs.

Agency costs consist of costs for construction, reconstruction, rehabilitation, repair, maintenance, engineering and administration. These costs are discussed in the third subsection below.

User costs include costs relating to travel time and delay, vehicle operation, accidents, discomfort and aesthetics. These costs may be viewed as the user costs resulting from the provision of a somewhat deteriorated pavement (instead of a new pavement); or they can be viewed as benefits to the user resulting from the provision of a road – benefits which decline somewhat as pavement condition deteriorates. The latter approach provides a more comprehensive picture of the benefits of highways; however, it creates some analytic complexities and it contributes little to the understanding of pavement LCCs. Accordingly, pavement LCC analyses usually focus on the costs to the user of pavement deterioration and on the additional user costs resulting from various types of pavement restoration activities. This will be our approach.

Analyses of the effects of pavement condition on routine user costs generally are limited to the effects on the costs of travel time and delay and on vehicle operating costs. These effects are discussed in the fourth subsection below. There has been little or no research into the effects of pavement condition on safety, though the safety costs of poor pavement probably are outweighed by the safety benefits of resulting reductions in speed. The effects of poor pavement on comfort and aesthetics are considered to be minor and their direct costs usually are ignored in comprehensive analyses of pavement costs, though the effects of decreased comfort on speed are reflected in the analyses of the effects on travel time.

We also plan to analyze the nonroutine costs of extra delay resulting from temporary road-use restrictions required to allow pavement reconstruction and resurfacing (but probably not other maintenance activities). These costs are discussed in the fifth subsection below.

External costs of road use include societal costs due to noise and to the emission of pollutants and greenhouse gases, all of which tend to increase as a result of pavement deterioration. However, these effects are ignored in most analyses of pavement LCCs, and we do not plan to address them in our study.

This section also includes a summary of some of the assumptions made by several states in their own LCC analyses of pavement costs (in the first subsection), and of discount rates used in these analyses (in the second subsection).

Life-Cycle Cost Analyses

A recent survey of state highway agencies (SHAs) obtained information about the life-cycle analyses conducted before constructing or reconstructing pavement (Cole, 1995). A somewhat simplified summary of the results of this survey is presented in Table 4.1. As the table indicates, each state uses a standard time period for its analyses, though the period used by the responding states varies between 30 and 50 years.

During the selected analysis period, each state assumes that a specified set of maintenance and repair activities will be undertaken according to a fixed schedule. Separate schedules are specified for rigid and flexible pavements, and some states also specify separate schedules on the basis of the traffic volume or location.

For flexible pavements, most of the schedules assume one to three overlays (with or without milling) during the analysis period, and they may also include some lesser actions (such as patching or sealing cracks). At one extreme, the Illinois analysis assumes five overlays on roads with particularly high pavement loads; at the other, for low-volume roads, Michigan assumes only surface treatments, patching and crack sealing.

Only nine of the 18 states listed in Table 4.1 assume that rigid pavement will be overlaid during the analysis period, three of these states only assume overlays for certain road types, and only Wisconsin assumes more than one overlay in the course of the analysis period (which is 50 years for Wisconsin). Most states also assume that joints will be rehabilitated one or more times during this period, and some assume a limited amount of slab replacement. Patching is also assumed in the analyses performed by several of the states.

The Discount Rate

The results of LCC analyses are influenced by the discount rate used. A high discount rate results in analyses that place a much lower value on future expenditures and benefits than on current expenditures and benefits. Thus, a high discount rate implies that the future costs of underdesigning pavement are relatively mild, while a low discount rate treats these costs as more significant. Hence, *ceteris paribus*, LCC analyses using low discount rates will support stronger pavement designs than corresponding analyses using higher discount rates.

The Office of Management and Budget (OMB) (1992, Section 8b) believes that benefit-cost analyses of public expenditures should include a “base-case” analysis in which a real discount rate of seven percent is used. The rationale for this relatively high rate is that “public investments … displace both private investment and consumption” and seven percent “approximates the marginal pretax [real] rate of return on an average investment in the private sector in recent years.”

Table 4.1 Life-Cycle Cost Analysis Periods Used by Various States

State	Period (Years)	Number of Overlays: Flexible Pavement	Number of Overlays: Rigid Pavement
California	35	2	1
Colorado	30	2	0-1
Florida	40	2	0-1
Georgia	30	2	-
Illinois	40	1-5	-
Iowa	30	1-2	0-1
Michigan	35	0-2	-
Minnesota	35	2	-
Nebraska	50	2	1
New Mexico	30	1	-
New York	33	2	1
North Carolina	30	2	-
Pennsylvania	40	4	1
South Carolina	30	2	-
Tennessee	30	2	1
Washington	40	3	-
Wisconsin	50	3-4	1-2
Wyoming	30	2	1

Source: Cole, 1995.

On the other hand, nearly all states use lower discount rates. A recent survey (Cole, 1995) found that, of 34 responding SHAs that perform LCC analyses, all but six used discount rates that were between three and five percent, with nearly half using four percent. (Two of the six other SHAs did not use any discount rate, two used variable rates, one used two percent, and one used seven percent). The lower discount rates used by the states are, at least in part, a reflection of a relatively low perceived cost of capital. Since the interest paid on state debt generally is exempt from Federal income taxes, interest rates paid by state agencies are lower than those paid by the Federal government or the private sector. However, this comparison ignores the cost of reduced income-tax receipts that state debt imposes on the Federal government.

On the basis of the above discussion, we plan to perform all LCC analyses in this study using discount rates of four percent and seven percent.

Agency Costs

For rigid pavements, agency costs consist of costs for constructing or reconstructing the pavement, flexible overlays, and contracted and in-house maintenance activities (patching, slab replacement, rehabilitation of joints, etc.).

Most states have a set of simple rules for obtaining preliminary estimates of the costs of pavement reconstruction and overlays (e.g., Mroczka, 1997, Nebraska, 1996, and Sebaaly, et. al., 1996); however, these rules reflect individual state experience and are not consistent across states. For the current study, we propose using cost data derived primarily from R. S. Means (1996) data for 1997. For Portland cement concrete (PCC), we estimate the cost per lane-mile, C_{PC}, as a function of pavement thickness, d:

$$C_{PC} = 3{,}000 + 22{,}000\ d \qquad (4.1)$$

Similarly, for asphalt concrete (AC), we estimate the cost per lane-mile, C_{AC}:

$$C_{AC} = 4{,}000 + 13{,}000\ d \qquad (4.2)$$

The cost estimates in Equations 4.1 and 4.2 reflect Means' estimates of costs for material, labor, equipment, and overhead and contractor profit, plus an additional three percent allowance for engineering costs (from Mroczka, 1997). Materials account for about 80 percent of total costs for PCC pavements and about 70 percent of total costs for AC pavements.

For the purpose of this study, it would appear appropriate to assume that total life-cycle maintenance costs incurred during the life of a pavement are not significantly affected by pavement life; i.e., total maintenance costs during the relatively short life of an underdesigned pavement are (approximately) the same as they would be over the longer life that would have resulted if the pavement had not been underdesigned. In effect, we are assuming that the same maintenance activities would occur in both cases, but, in the case of an underdesigned pavement, these activities would occur on a relatively accelerated schedule. This assumption appears appropriate for patching and selective slab replacement, though it may be less appropriate for joint rehabilitation.

Estimates of life-cycle maintenance costs are less readily available than other cost estimates required for this study. We have just ordered a recent study by Michigan State University (1998) that we have been told may contain useful data for our purposes. If this is not the case, we will probably develop an estimate of typical life-cycle maintenance costs using information on typical life-cycle maintenance activities assumed by Illinois, Michigan or Pennsylvania (as reported in Cole, 1995) and separate estimates of the costs of these activities.

Routine User Costs

As stated earlier, analyses of the effects of pavement condition on routine user costs generally are limited to the effects on travel time and on vehicle operating costs.

In the United States, virtually all estimates of the effects of road characteristics on vehicle operating costs are based on the results of a major study by Zaniewski, et al. (1982). This study developed estimates of the effects of speed, speed changes, curves, and pavement condition on fuel consumption, oil consumption, tire wear, maintenance and repair, and vehicle depreciation for eight types of highway vehicle. The effects of pavement condition were estimated as factors that would increase four of the five consumption rates (all but fuel consumption) as pavement condition deteriorates.

The most current version of the Zaniewski results is incorporated into FHWA's Highway Economic Requirements System (HERS) (Volpe National Transportation Systems Center [VNTSC], 1998, Section 7.1.2 and Appendix C). HERS incorporates a set of equations that have been fit to the original Zaniewski tables along with a set of adjustments to reflect the effects of inflation and changes in technology. The current version of HERS produces operating-cost estimates reflecting 1995 conditions and costs, and price indexes to produce cost estimates in 1997 dollars have also been used (Cambridge Systematics, 1998). It would be fairly straightforward to apply HERS to Highway Performance Monitoring System (HPMS) data (FHWA, 1993) for the rural principal arterial system to develop estimates of the average effect of pavement deterioration on operating costs for roads in this functional system.

HERS also produces estimates of the effects of pavement condition on speed, travel time, and travel-time costs. The effect on speed is estimated using a piecewise linear function[5] (VNTSC, 1998, Section 6.1.1.2) for the maximum speed (VROUGH, in mph) that can be traveled as a function of the pavement's present serviceability rating (PSR):

$$\text{VROUGH} = \begin{cases} 5 + 15\,\text{PSR} & \text{PSR} \leq 1.0 \\ 20 + 32.5\,(\text{PSR} - 1) & \text{PSR} > 1.0 \end{cases} \qquad (4.3)$$

Costs per vehicle-hour are estimated to range from \$14.30 (for small autos) to \$31.58 (for combination trucks with five or more axles). Estimates of the effect of pavement deterioration on travel-time costs can be developed using HERS at the same time as estimates of the effect on operating costs are developed.

[5] Other sources have estimated this effect using a concave downward function (FHWA, 1987) or a concave upward function (Watanatada, Dhareshwar and Rezende Lima, 1987, based on Brazilian data).

User Delay Due to Construction

Witczak and Rada (1984) have developed a relatively simple set of equations for estimating the cost of delay from the application of flexible overlays as a function of overlay thickness and traffic volume. For roads with shoulders,[6] the general form of their equation is:

$$C_{cd} = \frac{a \times NL \times ADT \times d_{OL.} \times PSI^{b}}{\log(ADT) - 1} \qquad (4.4)$$

where

C_{cd} = cost of construction delay, in 1984 dollars per mile;

NL = number of lanes;

ADT = average daily traffic;

d_{OL} = overlay thickness, in inches;

PSI = present serviceability index before resurfacing;

and a and b are parameters with values shown in Table 4.2.

For the purpose of our analyses, we shall assume that ADT during the period of resurfacing is equal to AADT, and we shall convert from 1984 dollars to 1997 or 1998 dollars using Bureau of Labor Statistics data on average earnings of civilian workers. We shall then apply Equation 4.4 to HPMS data on AADT and numbers of lanes for rural other principal arterials in order to produce a simple formula for the average cost of delay to overlays on these roads as a function of overlay thickness.

[6] Witczak and Rada develop a more complex equation for two-lane roads without shoulders.

Table 4.2 Parameters for Use in Equation 4.4

	a	b
Rural Roads		
2 Lanes	0.06015	0.661
≥ 4 Lanes		
ADT ≤ 22,500 NL - 45,000	0.02975	0.839
ADT > 22,500 NL - 45,000	0.577	0.093
Urban Streets and Roads		
2 Lanes	0.00954	1.511
≥ 4 Lanes		
ADT ≤ 25,500 NL - 51,000	0.02132	0.972
ADT > 25,500 NL - 51,000	0.243	0.184

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